DESIGN OF A 70 FOOT THRU PLATE GIRDER RAILROAD BRIDGE

BY

G. D. LEWIS

ARMOUR INSTITUTE OF TECHNOLOGY

1917

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A design of a 70 foot thru
 plate girder railroad

A DESIGN OF A 70 FOOT THRU PLATE GIRDER RAILROAD BRIDGE

A THESIS

PRESENTED BY

GEORGE DRAPER LEWIS

TO THE

PRESIDENT AND FACULTY

OF

ARMOUR INSTITUTE OF TECHNOLOGY

FOR THE DEGREE OF

CIVIL ENGINEER

MAY 31, 1917

APPROVED:

Alfred E. Phillips
Professor of Civil Engineering

Dean of Engineering Studies

Dean of Cultural Studies

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The Design of a 70' Through Plate Girder
Railway Bridge, being a Thesis
presented by
GEORGE D. LEWIS,
for the Degree of Civil Engineer.

27185

DESIGN OF THE SUPERSTRUCTURE.

General Notes

The bridge is to replace a pile trestle which carries a single steam railroad track over an electric line having a 60' right-of-way.

The track on either approach to the present bridge is on fairly heavy grades and therefore it will be impracticable from an operating standpoint to put in a bridge which will necessitate a steeper climb to reach the level of the bridge. The present vertical clearance, 21 feet, will have to be maintained; that is, the distance from the top of rail on the electric line to low steel of the overhead bridge cannot be decreased, so we will have to use a floor system which will allow us to retain this clearance. It will also be necessary to have the clear span of the permanent structure at least as wide as the right-of-way of the electric line. So it is easily shown that a 70' span will be very close to that actually required, and hence a 70' Through Plate Girder span will be used.

Loading

The live load will be Cooper's Class E55, or a concentrated load of 66,000# on each of two axles spaced 6' center to center.

The dead load shall consist of the weight of the structure with that of the deck. The weight of the deck is taken at 550# per lineal foot of bridge.

A static equivalent, to be called impact hereafter, shall be added to the computed stresses from dead and live load and shall be determined by the formula $I=L^2/L+D$ where I=impact, L=live load stress, and D=dead load stress.

As this bridge is on a tangent there will be no centrifugal force.

Deck

The deck will consist of 8"x8"x12' ties spaced on 13" centers, two lines of 10"x12" guard timbers placed at the ends of the ties, 90# rails, and four lines of 4"x10" guard rails placed just inside and outside of the rails.

Design of stringers

We will investigate 12" I-beams as stringers, placing two under each rail.

The end shear is as follows:

D. L.	700#
L. L.	18,300#
I.	17,600#
Total	36,600#

The live load end shear is found to be a maximum under the special loading. The bending moment is as follows:

D. L.	13,200 "#
L. L.	334,100 "#
I.	321,700 "#
Total	669,000 "#

From our specifications we find that the allowable stress in the compression flange of any member is found by the formula $S=16000 - 40^{1}/r$ where S= allowable unit stress, I=maximum unsupported length in inches and r=the radius of gyration about a vertical axis through the center of gravity. Thus we find that the allowable unit stress in the top flange of the stringers is 12,900# and it follows from this that the required section modulus per beam equals

669,000 ÷ 12,900 or 51.8.

The section modulus of a 12" I-beam at 55# is 53.5. So we will use four 12" I-beams at 55# for stringers and space them on 2'6" centers as seen by referring to the superstructure plan.

Design of floor beams.

For the end floor beam we will assume a section made up of a web plate, 18"x5/8", four angles 6"x6"x 5/8" and four cover plates 14"x1/2".

The end shear is found to be 91,500# as tabulated on the superstructure sheet, and from this we see that the web plate assumed will answer, as we only need 9.15 square inches to take care of this maximum shearing value.

The maximum bending moment figures out as follows:

D. L.	19,900 '#
L. L.	285,000 '#
I.	266,500 '#

To get the flange stress, we divide the values by the effective depth of the assumed section, that is, by the distance between the centers of gravity of the flanges.

The positions of the centers of gravity of the flanges are 1.625" from the backs of the outstanding legs of the flange angles and thus we get 17.00" for our effective depth.

The flange stresses are as follows:-

D. L.	14,000#
L. L.	201,000#
I.	188,000#
Total	403,000#

This requires a net area of 25.2 square inches for the bottom flange.

The unit stress for the top flange is found by getting the radius of gyration of the flange section about a vertical axis through the center of the web and then substituting in the formula $16000 - 40^{1}/r$ referred to under the stringer design. The value of r is found to be 3.47 and upon substitution we get 15,300# for the allowable unit stress for the top flange, which gives 26.4 square inches as the required gross area for the top flange.

The deduction for rivet holes is covered by the following clause from the specifications:

"The net section of a riveted tension member is

the sum of the net sections of its component parts. The net section of each part is determined separately and shall be the least area obtained by deducting from the gross area:

(1) The total area of all holes on any section perpendicular to the axis of the member, or,

(2) The total area of all holes on any zigzag section passing through those holes whose distance center to center is less than 1.25 g+h, where g=distance between gage lines and h=diameter of holes, which shall be taken as being 1/8" larger than nominal diameter of the rivets".

Also the specifications state that if there is no splice in the web plate then 1/8 of its area can be considered as effective flange area.

The flange sections finally decided on are given in tabular form on the superstructure sheet. And as the weight of the section does not differ materially from the assumed section no further calculations will be necessary.

To get the rivet pitch the formula p=sd/v is used, where p=the pitch, s=value of rivet, d=distance between centers between gage lines of the top and bottom flanges, and v=the shear to be transmitted. Thus for the pitch required to transmit the end shear we get by substitution 1.61" or 1-5/8". Between the

points where the outside stringers connect we will use a 3-1/2" pitch as this has been used with very good results on previous designs.

In connecting the floor beam to the girder, eight field rivets are required and seven field rivets are used to connect the stringers to the floor beams. In the first case the rivets are in double shear while in the second case they are in bearing on the web.

The design of the intermediate floor beam is taken up in the same way. The shears and flange stresses are given on the superstructure sheet together with the flange sections which will be used, and as they do not differ materially from the assumed section (See design of end floor beam) no further investigation is necessary.

The rivet pitch required to transmit the end shear is found to be 1-3/4". We will also use here the 3-1/2" pitch between the points where the outside stringers connect. We will need seven field rivets to connect the floor beams on to the girder.

The length of the cover plates are determined

by means of flange area diagrams, making them 3 feet longer than the required lengths as scaled.

It is a standard practice to make the corresponding cover plates on the floor beams the same length, hence we will use 11 feet for the lengths of the outside plates and 16 feet for the lengths of the inside.

Design of girder

From previous designs we can arrive fairly close at the section of the girder and from this we can base the estimate of the weight of the steel. The assumed section is made up of a web plate 90"x3/8", 4 angles of 8"x8"x3/4" and 4 cover plates 17"x5/8".

The estimated weight is:

Weight of girders,	58,000#
25% for girder details,	14,500#
Weight of floor beams,	40,500#
Weight of floor beam details,	10,000#
Weight of stringers,	15,400#
Weight of deck,	38,500#
Total	176,900#

which gives 1,260# per linear foot of girder.

The end reaction is:

D. L.	44,000#
L. L.	151,900#
I.	117,000#
Total	313,000#

The end shear is:

D. L.	35,500#
L. L.	121,800#
I.	93,700#
Total	251,000#

The values of the maximum bending mements are:

D. L.	772,000 '#
L. L.	2,267,000 '#
I.	1,692,000 '#

The effective depth of the assumed section is 7.375' (See design of end floor beam for method of deriving effective depth):

Thus for flange stresses we get:

D. L.	104,500#
L. L.	307,500#
I.	232,500#
Total	641,500#

and for the bottom flange we find that 40.1 square inches net area is required. As explained under the design of the end floor beam, we get 15,200# for the allowable unit stress for the top flange, and thus we require 42.1 square inches gross for this section.

For the top flange we will use:

2 angles 8"x8"x3/4"	22.48	sq.	in	gr.
1 plate 17"x5/8"	10.63	"	"	"
1 " 17"x1/2"	8.50	"	"	"
Total	42.01	"	"	"

and for the bottom flange we will use:

2 angles 8"x8"x3/4"	19.88	sq.	in.	net.
1 plate 17"x3/4"	11.25	"	"	"
1 " 17"x5/8"	9.38	"	"	"
Total	40.51	"	"	"

as the web plate is spliced we cannot, according to specifications, consider 1/8 of its area as effective flange area.

The lengths of the cover plates are determined by means of flange area diagrams making them 3 feet longer than the required lengths as scaled.

The inside cover plate on the top flange is run the full length of the girder. The required length of the inside cover plate on the bottom flange is found to be 53 feet, and the required length of the outside plate 35 feet. This last value is a little more than required for the corresponding plate on the top flange, but we will make it 35 feet long also.

The weight of the structural steel is now care-

fully estimated and is found to be 132,500# for the entire span. This together with the deck, 38,500#, forms the total dead load, and is 5,900# less than the value used in calculating. The revised values of end shear, end reaction, flange stress and required flange areas, are given on the superstructure drawing.

The shears in the different panels after the first are:

Second Panel:	D. L.	26,100#
	L. L.	99,000#
	I.	78,200#
	Total	203,300#
Third Panel:	D. L.	17,900#
	L. L.	80,000#
	I.	65,400#
	Total	163,300#
Fourth Panel:	D. L.	9,700#
	L. L.	59,250#
	I.	50,900#
	Total	119,850#

Substituting in the formula for rivet pitch (see floor beam design) we have for the required pitches:

First:-	2.625"
Second:-	3.235"
Third:-	4.03 "
Fourth:-	5.29 "

The specifications state that the maximum pitch on any gage line where a staggered pitch is used shall be 9 inches.

Design of splices.

The web plate will be spliced so that the rivets will be able to develope a stress 10% greater than the end shear. This requires 36 rivets, as they are in bearing on the 3/8" web plate. Using two rows, and the same number per row, as are in the stiffener angles, we get 34. We will use this number as one rivet, less will not make any material difference, and to punch extra holes it will be necessary to change the multiple punch. But as the same number of holes are used throughout, all stiffener angles, web splice plates, filler plates for same, etc., can be run through the multiple punch without any change in adjustment which makes the shop work much easier and saves a little time.

The flange angles are spliced near the ends so as to facilitate handling in the shop. The splice is located between stiffener angles in the end panels. In this distance we can get ten rivets through the vertical

legs and ten through the horizontal legs on each side of the splice. The rivets through the vertical legs are in double shear and in bearing on the web. The rivets through the horizontal legs are in double shear. Such a splice could develope 367,000# and we only require that it should develope the full strength of the member which is 22.88 x 15,200 or 347,800#.

Casting.

The end reaction is 313,000# and from the specifications we have that 500# per square inch is allowed in bearing of steel on masonry. Therefore, the base of the shoe must have an area of 626 square inches. And we will make the base 1'8" by 2'8". The shoe will be made 1'0" high.

All other parts not designed are detailed according to the authorized standards and practices of the Bridge Office of the Chicago and North Western Railway Company. The superstructure drawing is in accordance with the same standards and practices.

DESIGN OF SUBSTRUCTURE.

General Notes

One of the rules of good practice is that at no

point shall the thickness of an abutment be less than .45 of the distance of that point below the base of rail. By referring to the substructure drawing, it can readily be seen how the back is "stepped" down to make this rule hold. Also the width of the backwall at the level of the bridge seat shall be half of the distance that the bridge seat is below the base of rail.

Section of Center Line of Track.

A section is assumed and the position of the resultant is found to see whether or not it cuts the base within the "middle third".

The load from the span is assumed to be distributed equally over the footing of the abutment proper, and part of the wings. To find these parts we extend the center line of the footing of the abutment proper until it intersects the front page of the footings of the wings. Connect these points with the ends of the rear edge of the footing of the abutment. The resulting triangles are the parts referred to. Thus to get the amount of the load from the span per linear foot

of abutments, we divide the total load transmitted through the shoes by the length of the abutment plus the altitude of one of these triangles. In this case the total load per linear foot of abutment is found to be 61,000# as tabulated on the substructure drawing.

The next step is to find the location of the vertical component of the resultant of the dead and live load from the span together with the weight of the concrete and the weight of the earth backing. This is most readily done by taking moments. In this case we will take moments about the toe and they are found to be as follows:-

D. L. & L. L. from span,	32,000 '#
Concrete,	358,300 '#
Earth,	127,300 '#
Total	567,600 '#

Dividing this by 61,000#, we get 9.31' as the lever arm of this vertical force about the toe.

The value of the horizontal thrust of the earth backing is taken at 15 H^2, where H is the distance of the bottom of the footing below the base of rail. It is assumed to be applied two-thirds of H below the base

of rail. It is assumed to be applied two-thirds of H below the base of rail. This is probably a little strong.

The tangent of the angle between the horizontal thrust and the vertical component is found to be .251 which gives us 2.66' for the distance along the base between the points where the resultant and its vertical component cut the same. Thus we find that the resultant cuts the base at a distance of 6.65' from the toe which is well within the middle third.

Soil Pressures.

The average soil pressure is found by dividing the total load per linear foot of abutment by the width of the footing of same. This is found to be 3800#. An empirical formula used to get the maximum soil pressure at the toe is, (6n + 1) times the distance from the center of the base that the resultant cuts same divided by the base. In this case we get for 6n, 6 (8-6.64) ÷ 16 = .51. And thus we get 3800 x1.51 = 5700 for the value of maximum soil pressure.

Pile Pressures.

It is tried to keep the average pile pressure

around 16 tons. Knowing the load per linear foot of abutment, the total load to be supported by the piles in the area which is assumed to take care of the load from the span is readily calculated. After finding out what this value is, we design a pile layout which will keep the average pressure in this area about 16 tons per pile. It is not considered best to have the piles spaced closer than 2'9" between centers.

To find the average and maximum pile pressure, we take a strip near the center of track which will include two rows of piles if they are staggered and one row if they are not. Multiply the load per linear foot of abutment by the distance between the centers between the row, or rows of piling in the strip and the next rows and we have the total load to be supported by these piles. To get the average pressure per pile we divide this total load by the number of piles in the strip. In the layout given on the substructure sheet, it is found to be 32,700#.

To find the maximum pile pressure we find out the pressure due to excentricity, that is, due to the fact

that the resultant of all loads acting on the abutment does not intersect the base at the position of the center of gravity of the piles in the strip taken, and then add it to the pile pressure.

To find this value we first take moments to locate the position of the center of gravity of the piles in this strip. In this case we find the center of gravity to be 7.64' from the toe. We next find the moment of inertia of the piles about an axis through this point which is parallel to the edges of the footing. In this case it figures out to be 150.14. Then, dividing this by the distance of the center of the pile nearest the toe from this axis, the section modulus of this pile is obtained. In this case it is found to be 24.45. The distance between the center of gravity of the piles in this strip and the point where the resultant cuts, the base is determined. In this design it is 1.00'. From this we get the moment due to eccentricity which figures out, in this case, to be 295.000 '#. Dividing this by the section modulus determined above, the value of the increased pressure due to eccentric loading is found.

In this design it figures out to be 12,100#. And thus we get 44,800# for the value of the maximum pile pressure.

This method of finding the average and maximum pile pressures has been in use in the bridge office of the Chicago and North Western Railway Company for a number of years and has given very good results.

The substructure drawing as well as the design is in accordance with the standards and practices of the same office.

END SHEAR:
D.L. = 38,700#
L.L. = 121,800#
I = 92,200#
TOTAL = 252,700#

252,700 ÷ 10,000 = 25.3 □" GR. REQD.
WEB PL. 90"×3/8" = 33.75 □" GR.

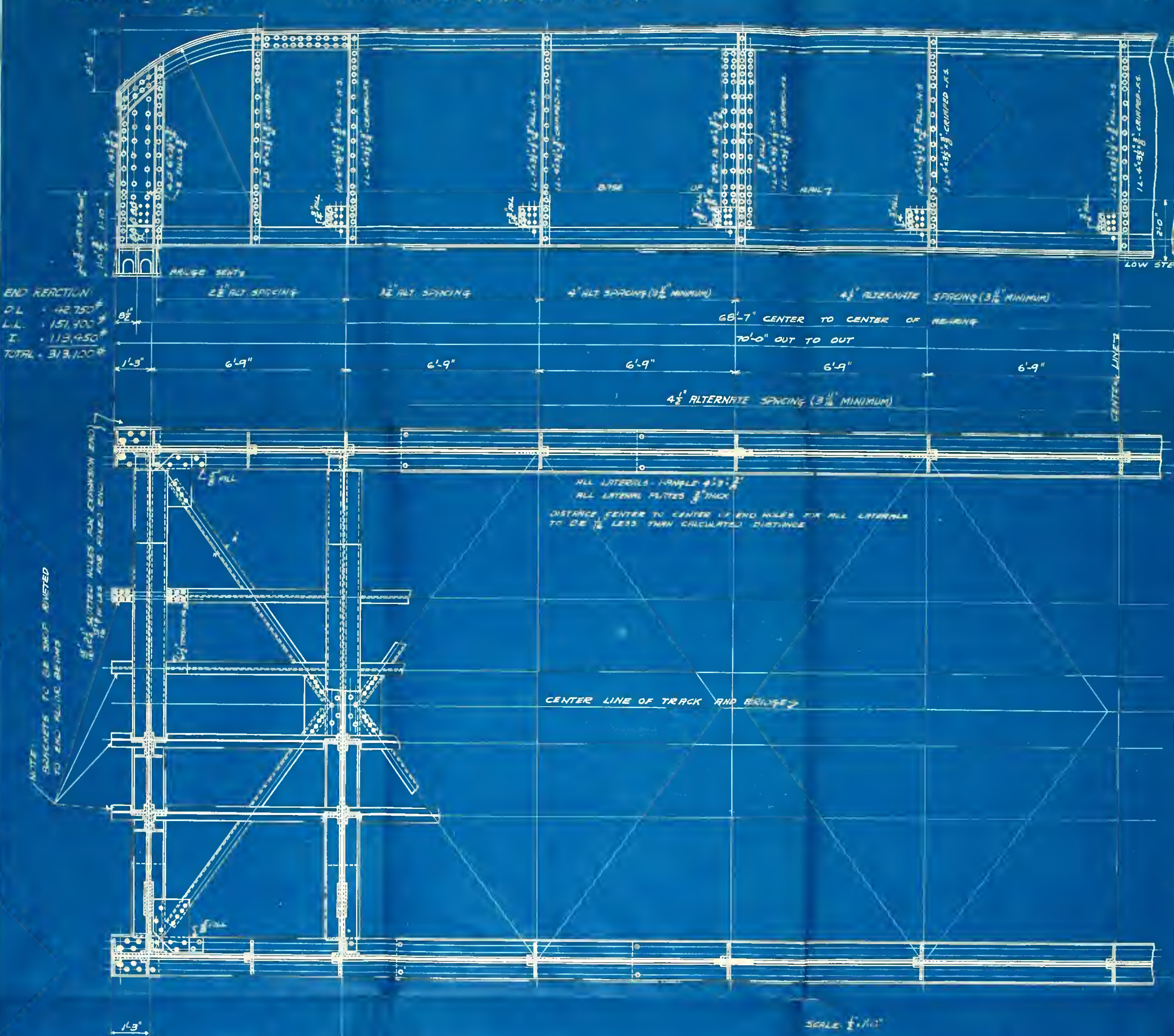
FLANGE STRESS
D.L. = 99,200#
L.L. = 307,300#
I = 232,500#
TOTAL = 639,000#

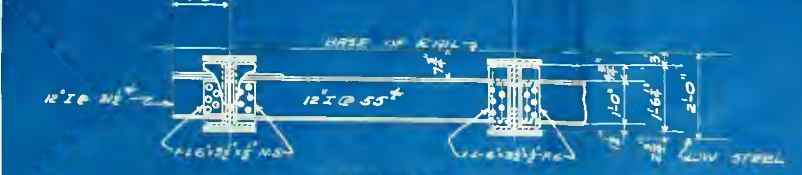
639,000 ÷ 16,000 = 39.9 □" NET REQD. FOR BOT. FLANGE
639,000 ÷ 15,200 = 42.1 □" GR. REQD. FOR TOP FLANGE

BOTTOM FLANGE:	
2∠ 6"×6"×5/8"	19.88 □" NET
1 PL. 17"×3/4"×53'	11.25 □" "
1 PL. 17"×5/8"×35'	9.38 □" "
	40.51 □" NET

TOP FLANGE:	
2∠ 6"×6"×1/2"	22.88
1 PL. 17"×5/8" FULL LG.	10.63
1 PL. 17"×1/2"×35'	8.50
	42.01

END REACTION:
D.L. = 42,750#
L.L. = 151,900#
I. = 118,450#
TOTAL = 313,100#


5'-0"
2'-3"
BRIDGE SEATS
2 1/2" ALT. SPACING
3 1/2" ALT. SPACING
4" ALT. SPACING (3 1/2" MINIMUM)
4 1/2" ALTERNATE SPACING (3 1/2" MINIMUM)
68'-7" CENTER TO CENTER OF BEARING
70'-0" OUT TO OUT
8 1/2"
1'-3"
6'-9"
6'-9"
6'-9"
6'-9"
6'-9"
CENTER LINE
4 1/2" ALTERNATE SPACING (3 1/4" MINIMUM)
BASE
BASE OF RAIL
LOW STEEL
2'-0"
3/8" FILL
1L-4"×3 1/2"×3/8"-CRIMPED-F.S.
ALL LATERALS - 1 ANGLE 4"×3"×3/8"
ALL LATERAL PLATES 3/8" THICK
DISTANCE CENTER TO CENTER OF END HOLES FOR ALL LATERALS TO BE 1/16 LESS THAN CALCULATED DISTANCE
NOTE: BRACKETS TO BE SHOP RIVETED TO END FLOOR BEAMS
13/16" × 2 1/4" SLOTTED HOLES FOR EXPANSION END
13/16" HOLES FOR FIXED END
CENTER LINE OF TRACK AND BRIDGE
SCALE 1/4" = 1'-0"



STRINGER

END SHEAR:		BENDING MOMENT	
D.L.	700#	D.L.	13,200'#
L.L.	18,300#	L.L.	334,100'#
I	17,400#	I	321,700'#
TOTAL	36,400#	TOTAL	669,000'#

ALLOWABLE UNIT STRESS FOR TOP FLANGE = 12,900#
669,000 ÷ 12,900 = 51.8 = SEC. MOD. REQD. PER BEAM
SECT. MODULUS OF 12"I @ 55# = 53.5

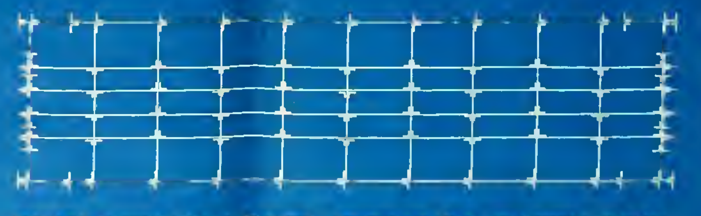
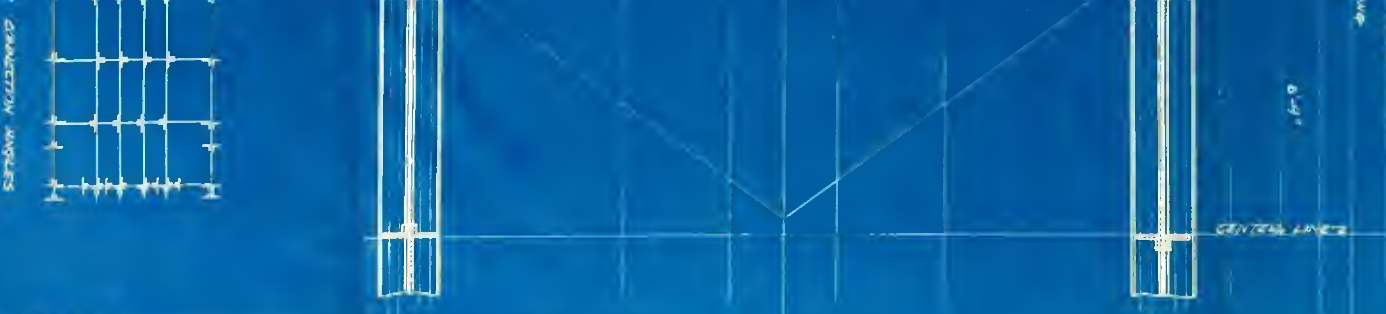
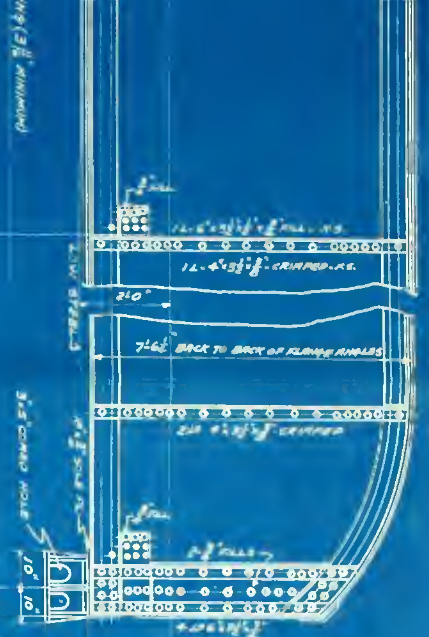


DIAGRAM SHOWING SHOP RIVETED STIFFENER AND CONNECTION ANGLES




1 L-6"×3½"×⅝" FILL - N.S.
1 L-4"×3½"×⅝" CRIMPED - F.S.
2'-0"
7'-6½" BACK TO BACK OF FLANGE ANGLES
2 L's 4"×3½"×⅝" CRIMPED
3"×5" CORED HOLE
4 L's 6"×6"×⅝"
8½"
10"
10"

TOP FLANGE:
2 L's 8×8×¾ 22.88 □" GR.
1 PL. 17×⅝ FULL LG. 10.63 □" "
1 PL. 17×½×35' 8.50 □" "
42.01 □" GR.

INTERMEDIATE FLOOR BEAM

END SHEAR:
D.L. 4600#
L.L. 41,800#
I. 37,700#
TOTAL 84,100#
84,100÷10,000=8.41 □" GR. REQ. WEB PL. 18"×⅝"=11.25 □" GR.

FLANGE STRESS:
D.L. 17,700#
L.L. 184,000#
I. 167,800#
TOTAL - 369,500#

REQ. AREA FOR BOT. FLANGE: 369,500÷16,000=23.1 □" NET
REQ. AREA FOR TOP FLANGE: 369,500÷15,300=24.1 □" GR.

BOTTOM FLANGE:
⅛ WEB - 1.41
2 L's 6×6×9/16" - 10.61 □" NET
1 PL. 14×7/16×16' - 6.00 "
1 PL. 14×7/16×11' - 5.26 "
23.28 □" NET

TOP FLANGE:
⅛ WEB 1.41
2 L's 6×6×9/16" 12.86 □" GR.
1 PL. 14×⅜×16' 5.25 "
1 PL. 14×⅜×11' 5.25 "
24.77 □" GR.

END FLOOR BEAM

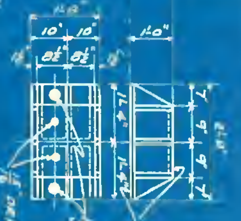
END SHEAR:
D.L. 3800#
L.L. 45,600#
I. 42,100#
TOTAL - 91,500#
91,500÷10,000=9.15 □" GR. REQ. WEB PL. 18"×⅝"=11.25 □" GR.

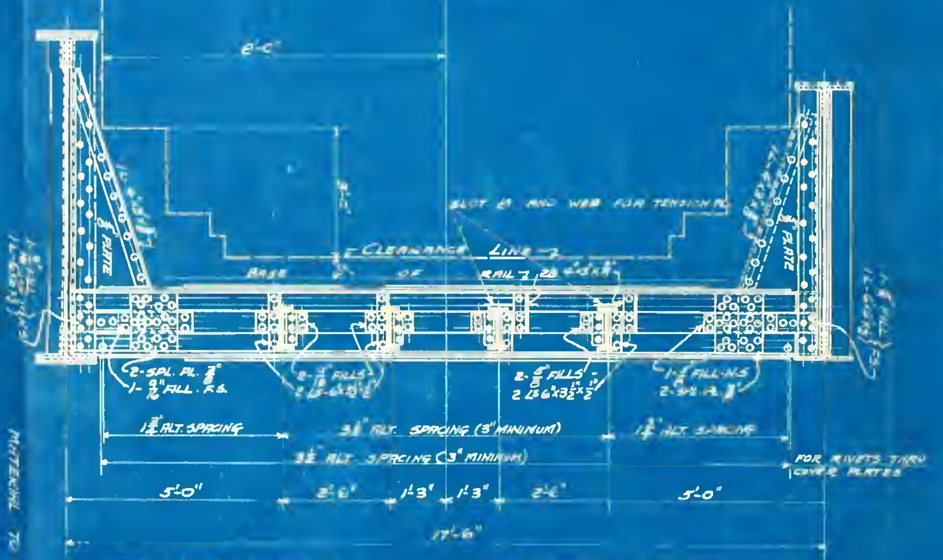
FLANGE STRESS:
D.L. 14,000#
L.L. 201,000#
I. 188,000#
TOTAL - 403,000#

REQ. AREA FOR BOT. FLANGE: 403,000÷16,000=25.2 □" NET
REQ. AREA FOR TOP FLANGE: 403,000÷15,300=26.4 □" GR.

BOTTOM FLANGE:
⅛ WEB 1.41
2 L's 6×6×⅝" 11.72 □" NET
1 PL. 14×⅝×16' 7.50 "
1 PL. 14×7/16×11' 5.25 "
25.88 □" NET

TOP FLANGE:
⅛ WEB 1.41
2 L's 6×6×⅝" 14.22 □" GR.
1 PL. 14×7/16×16' 6.13 "
1 PL. 14×7/16×11' 6.13 "
27.89 □" GR.


2'-8"
10"
10"
8½"
8½"
7"
9"
9"
7"
1'-4"
1'-4"
1'-0"
1⅛" DRILLED HOLES
1⅛" CORED HOLES FOR 1¼"×1'2" SWEDGED ANCHOR BOLTS WITH HEX. NUT AND 2"×2"×¼" WASHER
FLANGE



HALF SECTION SHOWING
INTERMEDIATE FLOOR BEAM

HALF SECTION SHOWING
END FLOOR BEAM

GENERAL NOTES

C.&N.W. RY. SPECIFICATIONS OF 1912
MATERIAL: SHAPES AND PLATES, O.H. STRUCTURAL STEEL; PEDESTALS, CAST IRON; RIVETS, RIVET STEEL, ⅞" DIAM.
OPEN HOLES: 15/16" DIAM. EXCEPT AS NOTED
REAMING: AS PER CHGO & NOR. WESTN. RY SPECIFICATIONS
PAINTING: " " " " " " "
WEIGHT: SHIPPING WEIGHT TO BE MARKED ON EACH PIECE
MARKS: ALL MARKS TO BE CLEAR AND DISTINCT
LOADING: LIVE LOAD, COOPER'S CLASS E-55
IMPACT = $\frac{L^2}{L+D}$, WHERE L = LIVE LOAD AND D = DEAD LOAD
UNIT STRESSES: TENSION, 16,000# PER SQ. IN. NET SECTION
BEARING, SHOP RIVETS, 24,000# PER SQ. IN. FIELD RIVETS, 20,000# PER SQ. IN.
SHEARING " " 12,000# " " " " " 10,000# "
" , WEB PLATES, 10,000# " " GROSS SECTION; 12,000# NET SECTION
COMPRESSION FOR TOP FLANGES: 16,000 - 40 $\frac{l}{r}$, WHERE l = UNSUPPORTED LENGTH AND r = HORIZONTAL RADIUS OF GYRATION

ESTIMATED WEIGHT OF MATERIAL FOR ONE SPAN:
STRUCTURAL STEEL: 132,300#
CAST IRON: 3,000#
TOTAL 135,300#

MATERIAL TO BE FURNISHED BY BRIDGE COMPANY:
1 - SPAN COMPLETE AS SHOWN FOR SINGLE TRACK.

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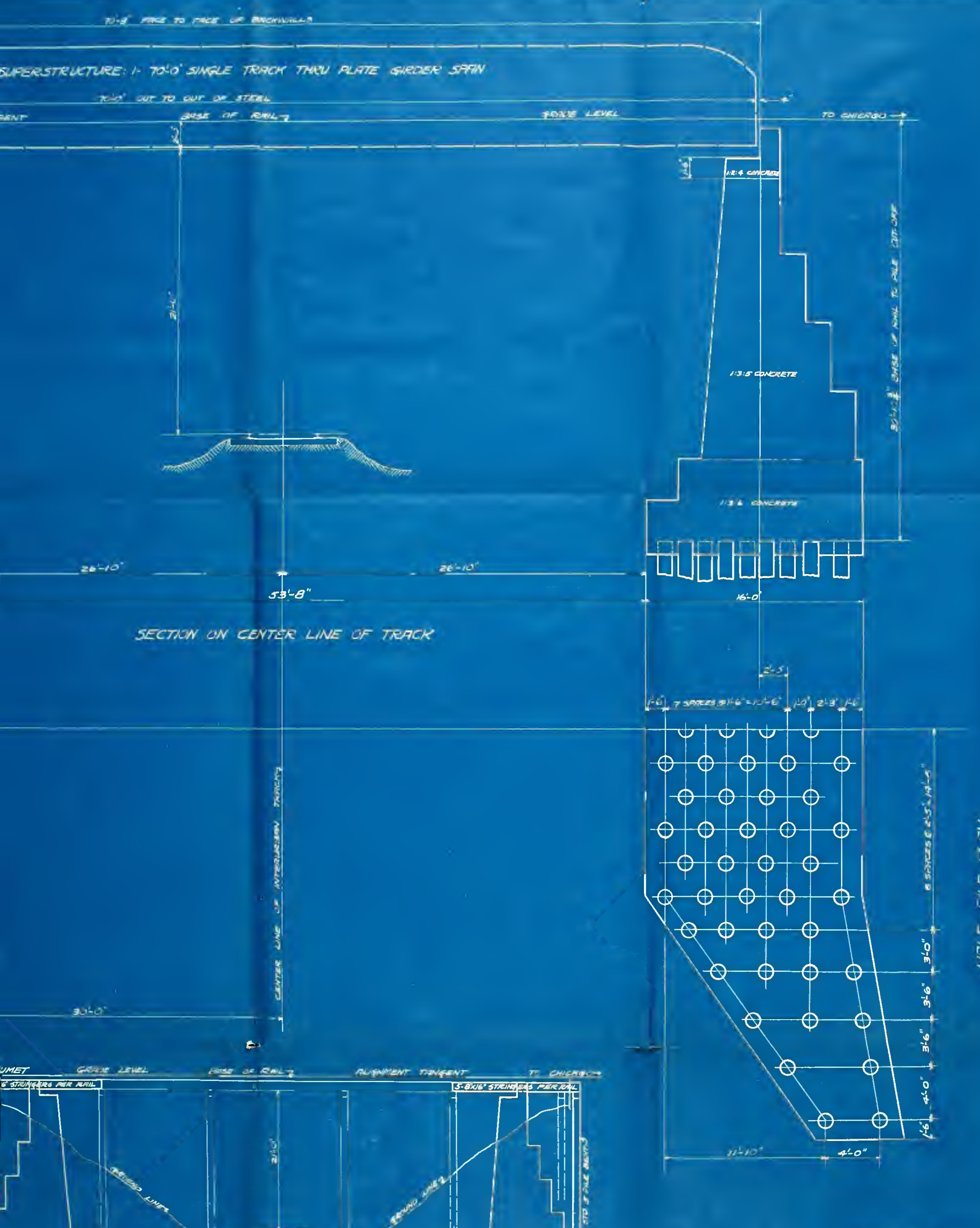
CIVIL ENGINEERING DEPARTMENT

A THESIS

PRESENTED BY GEORGE D. LEWIS
FOR THE DEGREE OF CIVIL ENGINEER

DESIGN OF A 70' S.T.-T.P.G. BRIDGE
SUPERSTRUCTURE
1917



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G.D.L. 3-10-17

NOTE:-
BACKWALLS TO BE BUILT TO FULL HEIGHT LEAVING OPENINGS FOR STRINGERS. CONCRETE BLOCKS ARE TO BE MADE TO FILL THESE OPENINGS AND SET IN PLACE AFTER STRINGERS ARE REMOVED

LOADS PER LINEAR FOOT OF ABUTMENT.

D.L. + L.L. FROM SPAN	= 11,000#
CONCRETE	= 40,600#
EARTH	= 9,400#
TOTAL	= 61,000#

AVERAGE SOIL PRESSURE	= 3800# PER SQ. FT.
MAXIMUM " "	= 5700# " " "
AVERAGE PILE "	= 32,700#
MAXIMUM " "	= 44,800#

QUANTITIES OF CONCRETE FOR TWO ABUTMENTS

19 CU. YDS. CONCRETE OF COMPOSITION	1:2:4
546 " " " " "	1:3:5
513 " " " " "	1:3:6
1078 " " TOTAL	

160 PILES REQUIRED

NOTE:- DIVISION ENGINEER TO DECIDE UPON NECESSITY OF USING FOUNDATION PILES